Traffic Testing Results from the FAA's National Airport Pavement Test Facility

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ABSTRACT

A brief history is given of traffic testing performed at the Federal Aviation Administration's National Airport Pavement Test Facility (NAPTF) since the facility was commissioned in 1999. Three construction cycles are described, with associated traffic test results and data analysis. The first construction cycle consisted of three rigid pavement test items and six flexible pavement test items. The second construction cycle consisted of a 100 ft by 40 ft (30.5 m by 12.2 m) rigid pavement test strip, a 15 ft by 15 ft (4.6 m by 4.6 m) free-standing test slab, and three 100 ft (30.5 m) long by 60 ft (18.3 m) wide rigid pavement test items. The third consisted of four flexible pavement test items in a 300 ft (91.4 m) long by 60 ft (18.3 m) wide test section. Results from the flexible pavement traffic tests have been incorporated in the FAA layered elastic flexible airport pavement thickness design procedure and for establishing new alpha factors for the CBR airport pavement thickness design procedure. Traffic testing of the second construction cycle is still in progress at the time of writing.

INTRODUCTION

The National Airport Pavement Test Facility (NAPTF) was commissioned on April 12, 1999 as a joint venture between the Federal Aviation Administration (FAA) and the Boeing Company. The primary objectives of the testing to be done at the facility were to: 1) provide additional traffic data for incorporation in new thickness design procedures under development by the FAA; 2) provide full-scale pavement response and failure information for use in airplane landing gear design and configuration studies; and 3) reexamine the CBR method of design for flexible pavements. All three of these objectives were established with particular reference to the level of pavement damage expected from the six-wheel landing gear on the B-777 airplane relative to the dual and dual-tandem landing gears used on aircraft in the remainder of the fleet.

The systems delivered with the facility consisted of a 900 ft (274.3 m) long by 60 ft (18.3 m) wide test pavement, embedded pavement instrumentation and a dynamic data acquisition system (20 samples per second), environmental instrumentation and a static data acquisition system (4 samples per hour), and a test vehicle for loading the test pavement with up to twelve aircraft tires at wheel loads of up to 75,000 lbs (334 kN). The initial test pavement consisted of three 300 ft (91.4 m) long sections with different subgrade strengths (low, medium, and high). Each subgrade had two flexible test items and one rigid test item constructed on it. This initial pavement is referred to as Construction Cycle One (CC1). Commissioning of the facility was followed by a 10-month period of verification, shakedown, and pavement response testing. Traffic testing started in February of 2000 with all of the test items loaded at 45,000 lbs (200 kN) per wheel. There were two traffic lanes; the "north" lane loaded by a six wheel triple dual tandem (TDT) configuration at 54 in (1,372 mm) dual spacing and 57 in (1,449 mm) tandem spacing; and the "south" lane loaded by a four-wheel dual tandem configuration at 44 in (1,118 mm) dual spacing and 58 in (1,473 mm) tandem spacing. Substantial corner cracking occurred in all three of the rigid pavement test items after 28 passes had been completed. Trafficking was halted to determine the cause of the early corner cracking. The cause was determined to be a large amount of concave upwards curling in the 20 ft by 20 ft (6.1 m by 6.1 m) slabs combined with the relatively small thickness of the slabs compared to normal airport construction. Trafficking on all of the rigid and flexible test items was continued in March of 2000 until the rigid test items were deemed failed. Trafficking then continued on the low- and medium-strength subgrade flexible test items until November of 2000 when ambient temperatures became too low for representative testing on the asphalt layers. Testing resumed in May of 2001 and stopped in September of the same year. The flexible test items were trafficked in a number of configurations and loads. The number of passes applied to the flexible test items varied from 12,000 to 52,000.

Because of the unsatisfactory results from the CC1 rigid pavements, a new construction cycle (CC2) was initiated with the ultimate goal of constructing and trafficking to failure three rigid pavement test items on different support structures (on-grade, aggregate subbase, and stabilized subbase) over the medium-strength subgrade. This construction cycle finally consisted of three phases: test strip, free-standing slab, and complete medium-strength test section. Testing was continuing on the medium-strength test section at the time this paper was prepared.

Completely satisfactory results had also not been obtained on the low-strength flexible test items and the 300 ft (91.4 m) long low-strength section was rebuilt from 2 ft (0.61 m) below the new subgrade surface with four conventional flexible test items. Target lives of the new test items varied from 200 to 36,000 passes. Subgrade strength was nominally 3 CBR. The construction cycle is called CC3. Six-wheel geometry during CC3 traffic tests was the same as for CC1, but the four-wheel loading was 54 in (1,372 mm) dual spacing and 57 in (1,449 mm) tandem spacing, the same as the six-wheel geometry.

FLEXIBLE PAVEMENT TEST RESULTS

The criteria used to determine failure of the flexible test items are the same as those defined in reference 1 for the multiple wheel heavy gear load (MWHGL) test series run by the U.S. Army Corps of Engineers, WES. These are: 1) one inch upheaval outside the traffic lane, signifying structural shear failure in the subgrade or other supporting layers; and 2) surface cracking to the point that the pavement is no longer waterproof, signifying complete structural failure of the surface layer.

The CC1 flexible pavement test items were constructed with both conventional support and with stabilized base support. The conventional structures had base courses consisting of 8 inches (203 mm) of FAA specification P-209, crushed aggregate base course, and the stabilized base structures had base courses consisting of 5 inches (127 mm) of FAA specification P-401, plant mix bituminous pavement. An equivalency of 1.6 therefore relates the base course layers. The subbase for the conventional structures consisted of FAA specification P-154, subbase course, and the subbase for the stabilized base structures consisted of P-209. The surface course in all cases was 5 inches (127 mm) of P-401. Table 1, which is taken from reference 2, shows the relevant properties of the CC1 flexible (and rigid) test items. The subgrade strengths are nominal and represent the targets during construction. All test results which are based on subgrade strength use a composite of measurements at various depths and positions made during construction and in trenches opened after trafficking. Details of the CC1 CBR measurements can be found in reference 2. Full structural failure did not occur in the low-strength CC1 test items, probably because the subgrade material contained a significant amount of silt and the upper layers of the subgrade dried somewhat over the long period of time between construction and starting the traffic testing. Subgrade strengths of approximately 5 CBR were measured after testing was completed instead of the 3 CBR design target. Therefore, results for only the medium-strength test items, which did show the expected subgrade shear failures, are reported.

The CC3 test items were all of conventional construction with the properties shown in figure 1. Figure 2 is a photograph of the CC3 test items after 19,500 passes on LFC3 and LFC4.

LFC1 and LFC2 had both failed by then. LFC1 is in the foreground of the photograph and is bounded by the first two white transverse paint lines. Transverse profiles, rut depth, and surface upheavals were measured at the three faint blue lines crossing the center of the test item. The vehicle was in motion when the photograph was taken, and was traveling away from the camera with the wheels descending to load LFC3.

Table 2 gives the wheel loads, total design thicknesses, measured subgrade strengths, and passes to failure for all of the CC1 and CC3 flexible test items included in the analysis. The NAPTF data consists of nine data points with wheel loads covering the range of 45,000 to 65,000 lbs (200 to 289 kN) and subgrade strengths covering the range 3.72 to 7.45 CBR.

Failure Model for Layered Elastic Based Design Procedure

In order to satisfy the FAA's requirement for updating the flexible pavement failure model in the LEDFAA computer program for airport pavement thickness design, a database of existing full-scale test data for heavy airplane wheel loads was assembled and the CC1 and CC3 test results added. The vertical strain at the top of the subgrade was then calculated for each data point using the LEDFAA structural model. Material properties were based on best estimates of the data provided in the reports of the full-scale tests. Pass-to-coverage ratio is calculated in LEDFAA for an equivalent wheel width projected onto the top of the subgrade and the same procedure must be used when constructing the LEDFAA failure model. Consequently, coverages to failure in LEDFAA are not the same as coverages to failure in the CBR method (discussed below).

Figure 3 shows the resulting values of vertical strain plotted against coverages to failure on a log-log scale. The NAPTF results match well with the existing data and the six-wheel data points do not show significant deviation from the rest of the data points (ideally, whatever the gear geometry, wheel load, and structural properties, all of the data points should fall on a single describing curve). The data points also indicate a change in characteristic slope at high coverage levels.

The dark line passing through the middle of the data points in figure 3 is the least squares curve fit to the data. The pair of straight lines marked "LEDFAA 1.3 Failure Model" is the failure model implemented in the version of the program specified for use in reference 3, and provides thickness designs with the LEDFAA structural and traffic models which are compatible with the thickness designs determined according to the conventional CBR procedures also described and implemented in reference 3. The second straight line in the LEDFAA failure model was introduced to maintain compatibility with the CBR method and to make allowance for the leveling off of the data points indicated by the figure. The second line also gives estimates of pavement thickness for extended pavement life which are compatible with the recommendations of reference 3. Further adjustment of this failure model is expected before the follow-on computer program FEDFAA is finalized, but not to the extent that thickness designs will be significantly changed.

Reconciliation with CBR ESWL Based Design Procedure

The thicknesses of the flexible test items for CC1 and CC3 were based on CBR and LEDFAA designs in which the design life was one tenth the target life. This was done partly because the trafficking was to be done indoors and the asphalt temperatures were unlikely to be as high as daytime temperatures at a field installation, and partly because it was assumed that the design procedures were conservative for the structures being built. This produced test item failure at close to the target number of passes for the medium-strength CC1 and the low-strength CC3 test items, but after computing alpha factors and plotting the NAPTF alpha factors alongside the MWHGL alpha factors, poor correspondence was found between the two sets of results. Resolving this discrepancy required conversion of the NAPTF structures to equivalent MWHGL structures.

When making a CBR design according to reference 3, the total thickness is first found based on an equivalent single wheel load (ESWL) computed using a uniform half-space (Boussinesq) structural model. The ESWL is then converted to a total thickness to the top of the subgrade based on the MWHGL derived alpha factors. For heavy aircraft, five inches (127 mm) of P-401 and a minimum of eight inches (203 mm) of crushed aggregate base course are required. The combined surface and base thickness is subtracted from the total thickness to find the subbase thickness and appropriate conversions made to obtain stabilized base and crushed aggregate subbase thicknesses. (This last step is not necessary for the NAPTF conventional structures.) But the MWHGL alpha factors were derived from full-scale tests on structures which had 3 inches (76 mm) of surface course and 6 inches (152 mm) of crushed aggregate base course. The NAPTF structures must therefore be converted to equivalent MWHGL reference structures in order for the alpha factors to be compared on a consistent basis.

Table 3 is a listing of the differences between the NAPTF and MWHGL structures and tables 4 and 5 give the thickness equivalency factors for converting NAPTF layers to equivalent MWHGL layers. Figure 4 illustrates the procedure for converting the structures using CC3-LFC1 as an example. Figure 4(a) is the original structure and figure 4(f) is the equivalent reference structure. Specific steps in the procedure are as follows:

1. Convert 2 inches (51 mm) of the asphalt surface to 3.2 inches (81 mm) of crushed aggregate by multiplying by 1.6. This leaves 3 inches (76 mm) of asphalt surface for the MWHGL equivalent structure. Figure 4(b).
2. Add the 3.2 inches (81 mm) of crushed aggregate to the existing 8 inches (203 mm) of crushed aggregate.
3. Subtract 6 inches (152 mm) of crushed aggregate for the MWHGL equivalent structure. Figure 4(c).
4. Convert the remaining 5.2 inches (132 mm) of crushed aggregate to 8.3 inches (211 mm) of standard quality subbase by multiplying by 1.6. Figure 4(d).
5. Convert 16 inches (406 mm) of high quality subbase to 19.2 inches (488 mm) of standard quality subbase by multiplying by 1.2. Figure 4(e).
6. Add 8.3 to 19.2 to give 27.5 inches (698 mm) of standard quality subbase. Figure 4(f).
7. The total thickness of the MWHGL equivalent structure is therefore 3 + 6 + 27.5 = 36.5 inches (927 mm). This is used to calculate the alpha factors for the two load cases on LFC1.

One further change was that the C5-A gear used in the MWHGL test was considered to be two six-wheel gears in tandem as opposed to a single twelve-wheel gear as was done in the MWHGL analysis. The effect is to reduce the ESWL, causing an increase in the alpha factor for the gear by about 10 percent. Pass-to-coverage ratio remains the same.

Figure 5 shows the MWHGL and NAPTF results plotted on the same chart with no conversion of the NAPTF structure layers to equivalent thicknesses, except for MFS which was converted to an equivalent conventional structure following the procedures in reference 3. The lines marked "Single Wheel," "Twin Tandem," and "12 Wheels" are the alpha factor curves originally defined by the MWHGL test data and incorporated in the flexible pavement design procedure of reference 3. The NAPTF data points clearly fall below the MWHGL results.

Figure 6 is the same as figure 5 except that the NAPTF structures have been converted to equivalent MWHGL reference structures. Much better correspondence now exists between the two sets of test results. This indicates that an extra degree of conservatism, in terms of protecting the subgrade from failure, has been built into the design procedure by increasing the minimum requirements for surface and base layer thicknesses and not reducing the total thickness to make allowance for the extra subgrade protection provided by the thicker upper layers. Supporting information and analysis for these results will be presented in a forthcoming publication together with a discussion of the effect on aircraft classification number (ACN) calculations.

RIGID PAVEMENT TEST RESULTS

First Construction Cycle

The CC1 rigid pavement test items were constructed according to normal FAA requirements and practices with the typical two-part aggregate and sand mix. Slab size was 20 ft by 20 ft (6.1 m by 6.1 m) and slab thickness varied from 9 inches (241 mm) to 11 inches (279 mm). Thin slabs were required in order to provide design lives within the time available for testing to failure. Concrete strengths were also high (about 1,000 psi (6.9 MPa)), another factor leading to thin slabs.

The FAA requirement for maximum slab size relative to thickness and support stiffness is based on temperature induced curling and excessive curling was not expected because the pavements were to be built and loaded indoors. However, significant curling did occur, leading to very early top-down corner cracking. Measurements made after construction showed that the temperature gradients in the slabs were about one tenth what would be expected under typical field conditions and that temperature related curling was insignificant compared to the amounts observed. The large amount of curling observed was eventually considered to have been caused by drying shrinkage and large vertical moisture gradients in the slabs in combination with the large, thin, slabs and, possibly, a mix prone to moisture shrinkage.

A detailed description of the CC1 rigid pavement responses and performance during this test series is contained in reference 4.

Second Construction Cycle: Test Strip

Before committing to another full-size construction project, it was decided to build a test strip to try to quantify the effects of the different variables on curling and pavement performance. The existing concrete slabs of LRS were removed and the surface of the existing econocrete cleaned. Eight 15 ft by 15 ft (4.6 m by 4.6 m) slabs and four 20 ft by 20 ft (6.1 m by 6.1 m) slabs were then constructed in two lanes. All slabs were 11 inches (274 mm) thick placed directly on the existing econocrete with a paper bond breaker. The CC1 mix was used in one of the lanes and an optimized, three-aggregate, mix was used in the other lane. Details of the geometric and mix designs are given in reference 5.

The longitudinal joints were doweled and the transverse joints sawed. The strip was cured for 30 days with a wet burlap covering. Corner and center-joint curling was measured during and after curing. Although significant curling was measured, of the same order as that measured in the CC1 pavements, there was no significant difference between the curling of the two lanes with different mix designs. The large slabs curled more than the small slabs, as expected.

After measuring the unloaded environmental responses of the test strip, and running a comprehensive series of tests to measure the response of the pavement to moving loads, traffic loading was applied with a dual-tandem configuration at 55,000 lbs (245 kN) wheel loads. The lateral wander pattern was adjusted so that the slab lane having the optimized mix was loaded by all four wheels, and with one tandem pair of wheels loading the inside edge of the longitudinal joint, when the gear was at the center of the wander pattern. Figure 7 shows a map of the cracks observed during the traffic loading, together with a numbered chronological listing of the observed occurrences of the cracks. The general pattern of crack occurrence in the 15 ft (4.6 m) slabs was that the first cracks to form were top-down corner cracks in the lane receiving only two-wheel loading. These were followed by transverse and longitudinal bottom-up cracks in the lane receiving four-wheel loading and further corner and longitudinal top-down cracks in the two-wheel lane. Trafficking was stopped at 800 passes on the 20 ft (6.1 m) slabs and at 5,000 passes on the 15 ft (4.6 m) slabs. Traffic testing on the test strip ended in April of 2002.

Based on these results, the 15 ft (4.6 m) slabs were considered to represent a viable design for obtaining full-scale traffic data matching the failure mode used in the FAA design procedure (bottom-up cracking due to fatigue from tensile stresses at the bottom surfaces of the slabs). The optimized mix design was not felt to provide any significant benefit in terms of improved curling behavior and would only result in higher strengths and therefore thinner slabs. Also, manipulating cement and water content was not successful in significantly reducing flexural strength so that thicker slabs could be used.

Second Construction Cycle: Free Standing Slab

Concrete flexural strength, at least in the short term, can be reduced by replacing some of the cement with flyash. Trial batches of a series of flyash mixes were made to determine the effect of flyash replacement on the strength of concrete made with local materials. Worthwhile reductions (to below 750 psi (5.2 MPa)) were obtained with flyash replacement of 50 percent or greater and a free-standing slab, 15 ft by 15 ft by 11 inches (4.6 m by 4.6 m by 279 mm) thick, with 60 percent flyash in the mix, was constructed on top of an existing concrete pavement to see if the high flyash content had any adverse effects not shown in the trial batch results. The slab was instrumented to measure internal temperatures, internal relative humidity, horizontal strains, and vertical deflections relative to the supporting slab. The slab was cured with wet burlap for 28 days.

Figure 8 is taken from reference 6 and shows vertical displacement at three of the corners of the slab and at one center-edge measured over a period of six and one half months starting at the time of placement. For the first 28 days, during the curing period, the slab stayed essentially flat. After removal of the burlap, moisture loss from the top of the slab caused rapid shrinkage at the top of the slab and consequent curling and uplift at the corners. The rate of shrinking reduced after a further 30 days, but the vertical displacements continued trending upwards until the wet burlap was replaced five and three quarters months after concrete placement. The vertical deflections then decreased rapidly for about ten days after which they settled slowly to a steady value. Daily cycles of temperature induced curling can be seen superimposed on the drying shrinkage induced curling. The temperature effects are small compared to the moisture effects. The maximum measured vertical displacement was 200 mils (5 mm), and the residual vertical displacement varied from 40 mils (1 mm) to 100 mils (2.5 mm). For application to test design, the most important features of these measurements are that the slab remained flat during curing, while the wet burlap remained in place, and that there was considerable residual curl after drying the surface and then replacing the wet burlap.

Second Construction Cycle: Test Items

Based on the preceding results, the following construction and test plan was prepared for trafficking rigid airport pavements to failure.

1. Three test items to be constructed on the medium-strength subgrade: econocrete subbase, aggregate subbase, and slab-on-subgrade.
2. Slab size of 15 ft by 15 ft (4.6 m by 4.6 m).
3. Slab thickness of 12 inches (305 mm).
4. Two-aggregate mix with 50 percent flyash replacement.
5. Longitudinal and transverse joints both doweled.
6. Wet burlap cure for 28 days.
7. Wet the surface of the test items frequently enough to keep total vertical curl at the corners of the slabs to 20 mils (0.5 mm) or less.
8. Traffic the north wheel path with a triple dual tandem configuration and the south wheel path with a dual tandem configuration.

Construction of the test items was completed on March 2, 2004, and testing started on April 27. At the time of writing, approximately 9,000 passes have been completed on the test items, with varying degrees of pavement deterioration observed. Top-down cracking has not been eliminated in the slab lane receiving two-wheel loading, but bottom-up longitudinal cracks have occurred in the slab lanes receiving four-wheel loading.

Figure 9 shows concrete being placed in one of the test item slabs and figure 10 shows a strain gage installation before concrete placement.

CONCLUSIONS

Two construction cycles, with associated traffic tests, have been completed at the NAPTF and another one is almost complete. Uniformity of construction and reliability of the apparatus for applying the loads to the pavements have been good enough to allow the results of the traffic tests to be incorporated in the FAA layered elastic flexible pavement design procedure. The test results are also being used to establish appropriate alpha factors for four- and six-wheel landing gears in the CBR method of flexible pavement design.

Most of the test results and analyses discussed in this paper are based on raw data, together with a great deal of other supporting information, which can be downloaded from the AAR-410 website at www.airporttech.tc.faa.com.

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TABLE 1. CC1 pavement cross-sectional details.

	Surface Layer		Base Layer		Subbase Layer		Subgrade		
Item ID*	Type	Thickness inch	Type	Thickness inch	Type	Thickness inch	Soil Type	CBR	Strength
LRS	P-501	11.0	P-306	6.13	P-154	8.38	MH-CH	4	Low
LFS	P-401	5.0	P-401	4.88	P-209	29.63	MH-CH	4	Low
LFC	P-401	5.13	P-209	7.75	P-154	36.38	MH-CH	4	Low
MFC	P-401	5.13	P-209	7.88	P-154	12.13	CL-CH	8	Medium
MFS	P-401	5.0	P-401	4.88	P-209	8.5	CL-CH	8	Medium
MRS	P-501	9.75	P-306	5.88	P-154	8.63	CL-CH	8	Medium
HRS	P-501	9.0	P-306	6.0	P-154	6.63	SW-SM	20	High
HFS	P-401	5.13	P-401	4.5	None		SW-SM	20	High
HFC	P-401	5.25	P-209	7.88	None	-	SW-SM	20	High

*Nomenclature
L, M, H = low-, medium-, high-strength subgrade
R, F = rigid, flexible pavement structure
S, C = stabilized base, conventional base

TABLE 2. Summary of NAPTF flexible pavement full-scale test results.

Wheel Configura-tion	Test Item	Wheel Load, lbs[1]	Repetitions to Failure	Design Thickness		Subgrade CBR[3]
				in	cm	
6-Wheel	CC3-LFC1	55,000	90	29	73.7	3.72
	CC3-LFC2	55,000	1,584	37	94.0	4.38
	CC3-LFC3	65,000	20,000	47	119.4	4.38[4]
	CC1-MFC	45,000	13,000	25	63.5	7.45
4-Wheel	CC3-LFC1	55,000	132	29	73.7	4.32
	CC3-LFC2	55,000	2,970	37	94.0	4.32
	CC3-LFC3	65,000	40,000[2]	47	119.4	4.32[4]
	CC1-MFC	45,000	12,000	25	63.5	7.34
	CC1-MFS	45,000	19,000	18.5	47.0	7.43

Notes to Table 1.

1. 45 kips = 200 kN, 55 kips = 244 kN, 65 kips = 289 kN.
2. Repetitions to failure for LFC3 – 4-wheel is from extrapolated rut depth curve.
3. CBR computed as the average of the following measurements: acceptance surface, trench surface, and trench pits 12 and 24 inches (30.5 and 61.0 cm) from the surface of the subgrade.
4. Trench not opened in LFC3. The CBR values for LFC3 have been given the same values as those in LFC2.

Table 3. Comparison between MWHGL and NAPTF structural properties.

Layer	MWHGL Conventional Pavements			NAPTF Conventional Pavements			NAPTF Stabilized Base Pavements		
	Material	Thickness		Material	Thickness		Material	Thickness	
		in	cm		in	cm		in	cm
Surface	Asphalt	3	7.6	Asphalt	5	12.7	Asphalt	5	12.7
Base	Crushed aggregate	6	15.2	Crushed aggregate	8	20.3	Asphalt	5	12.7
Subbase	Uncrushed aggregate	variable		Crushed stone screenings	variable		Crushed aggregate	variable	

Table 4. Legend for material identification.

AC	= Asphaltic Concrete = MWHGL and NAPTF (P-401) surface course material.
CA	= Crushed Aggregate = MWHGL and NAPTF (P-209) base course material.
SQS	= Standard Quality Subbase = MWHGL subbase course material.
HQS	= High Quality Subbase = NAPTF (P-154) subbase course material.

Table 5. Layer thickness equivalency factors.

CA	= 1.6 × AC
SQS	= 1.6 × CA
SQS	= 1.2 × HQS

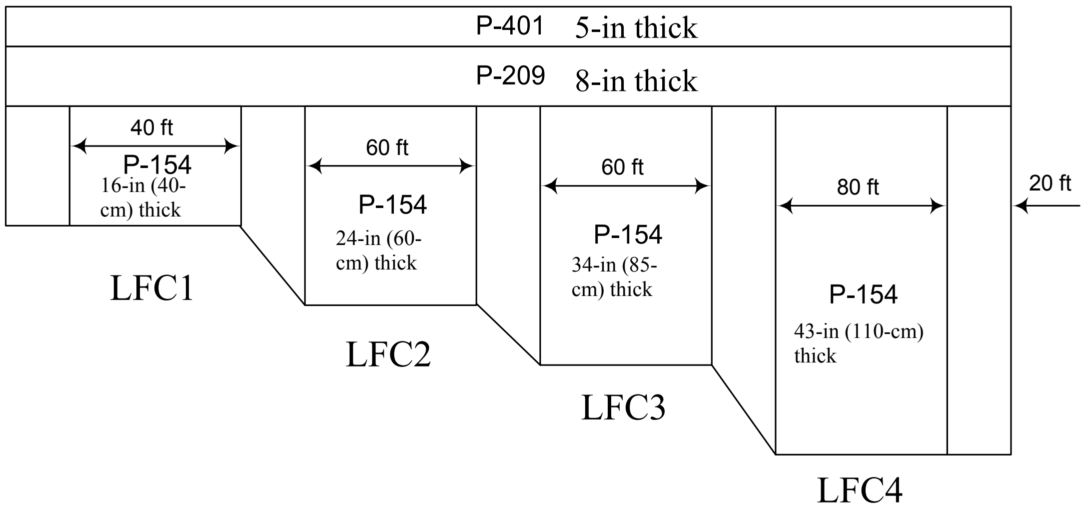

FIGURE 1. Cross-sectional layout of the LFC-1 through LFC-4 pavements.

FIGURE 2. CC3 flexible test pavement at 19,500 passes.

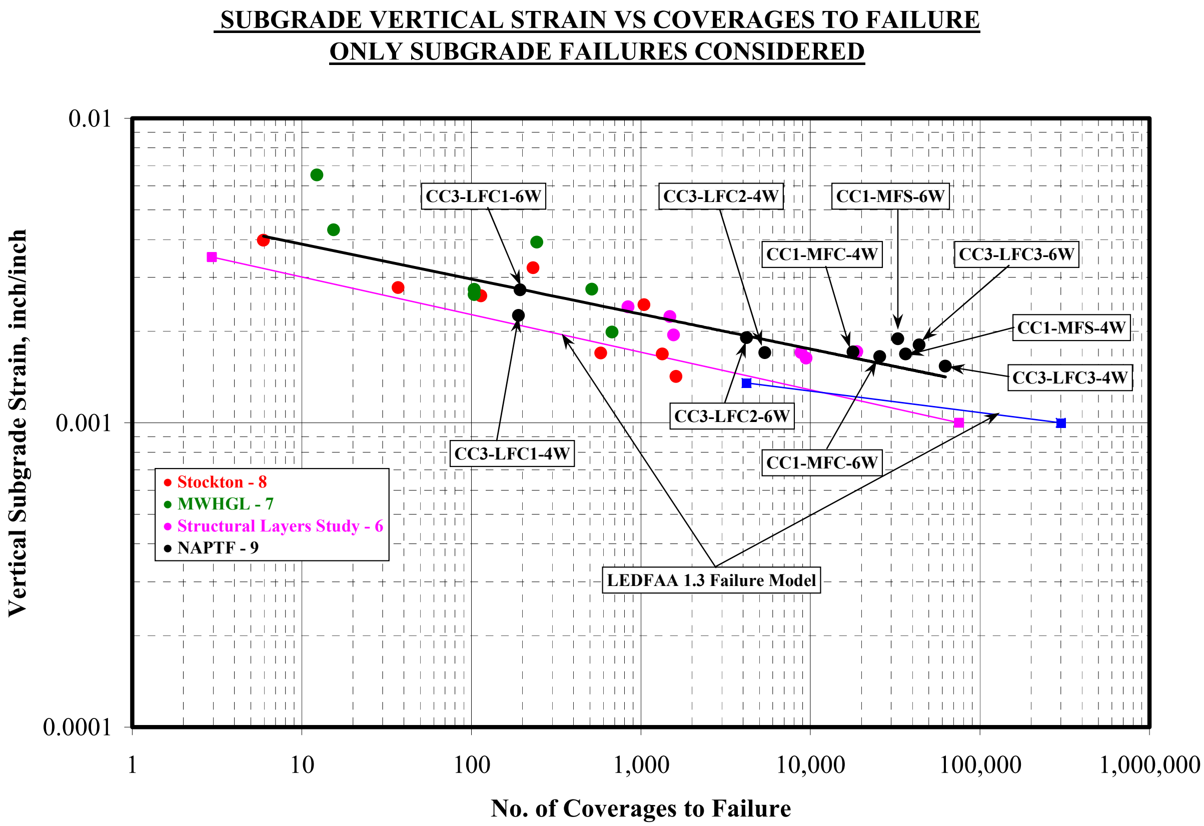

FIGURE 3. LEDFAA failure model for all available full-scale test data points with reported subgrade shear failure.

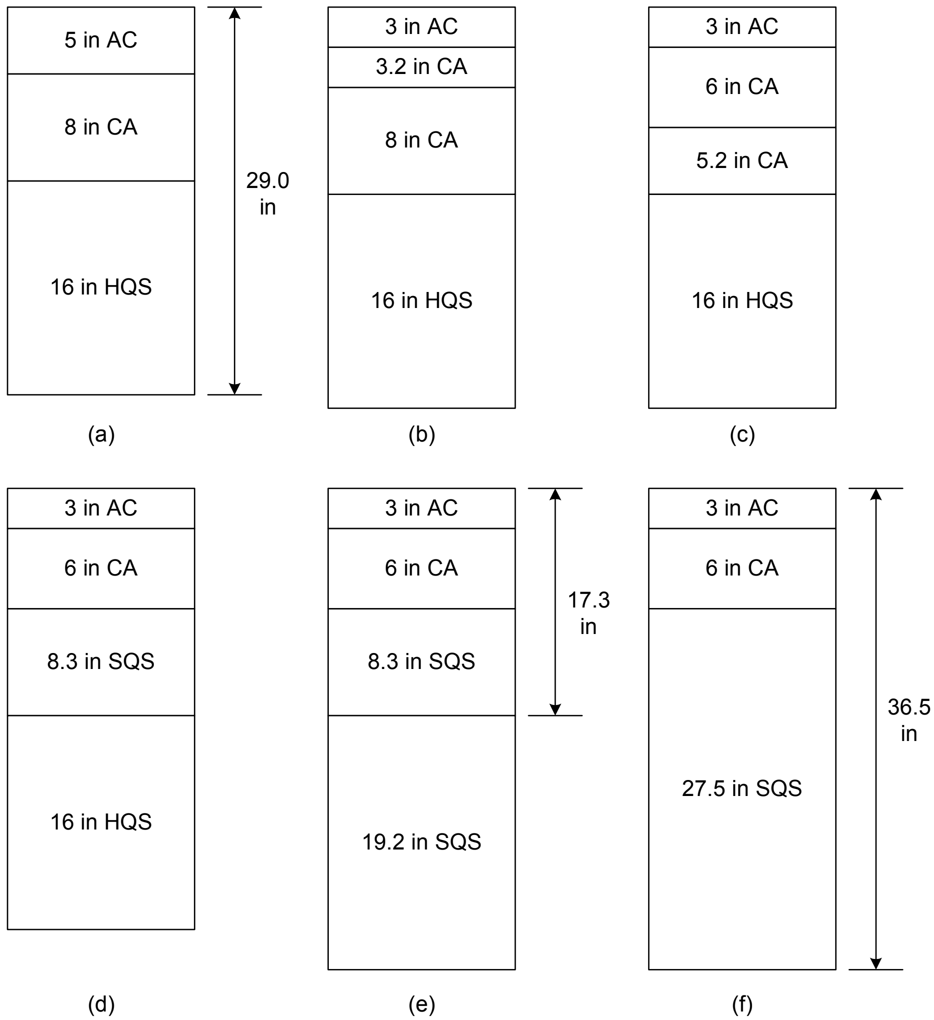

Figure 4. Procedure for converting NAPTF structures to equivalent MWHGL structures. 1 in = 2.54 cm.

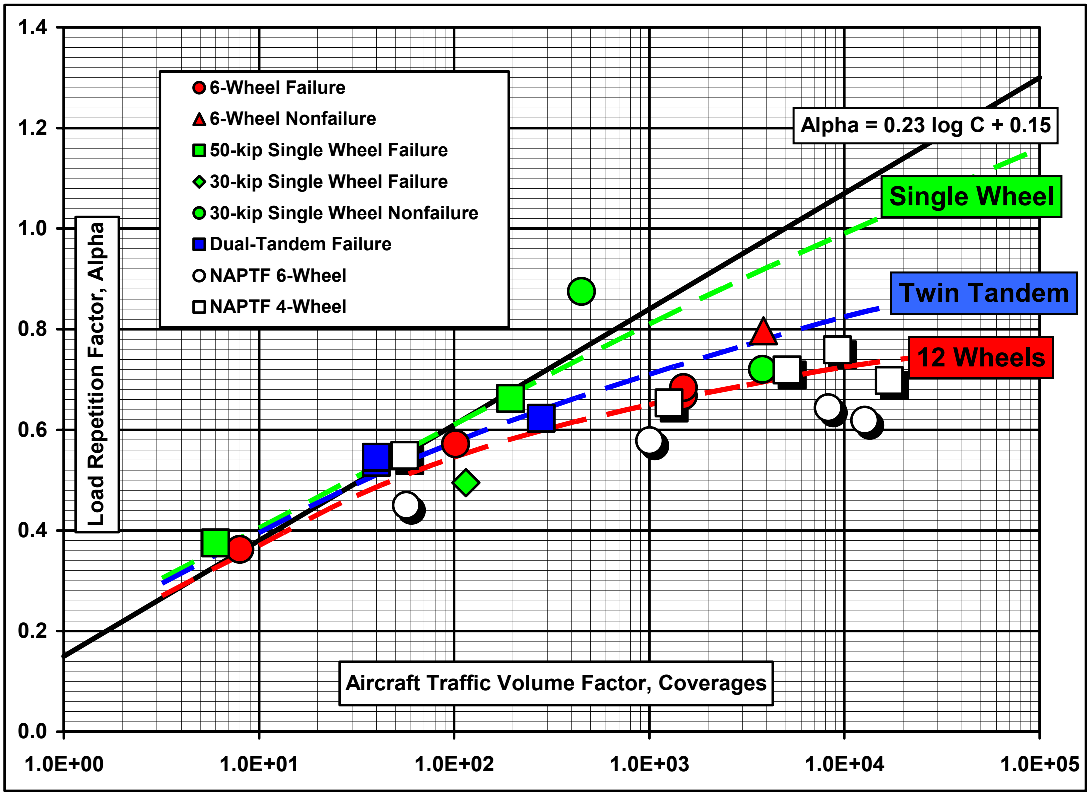

Figure 5. NAPTF and MWHGL alpha factor results with C5-A as a six-wheel gear and without conversion of NAPTF structures to equivalent MWHGL reference structures.

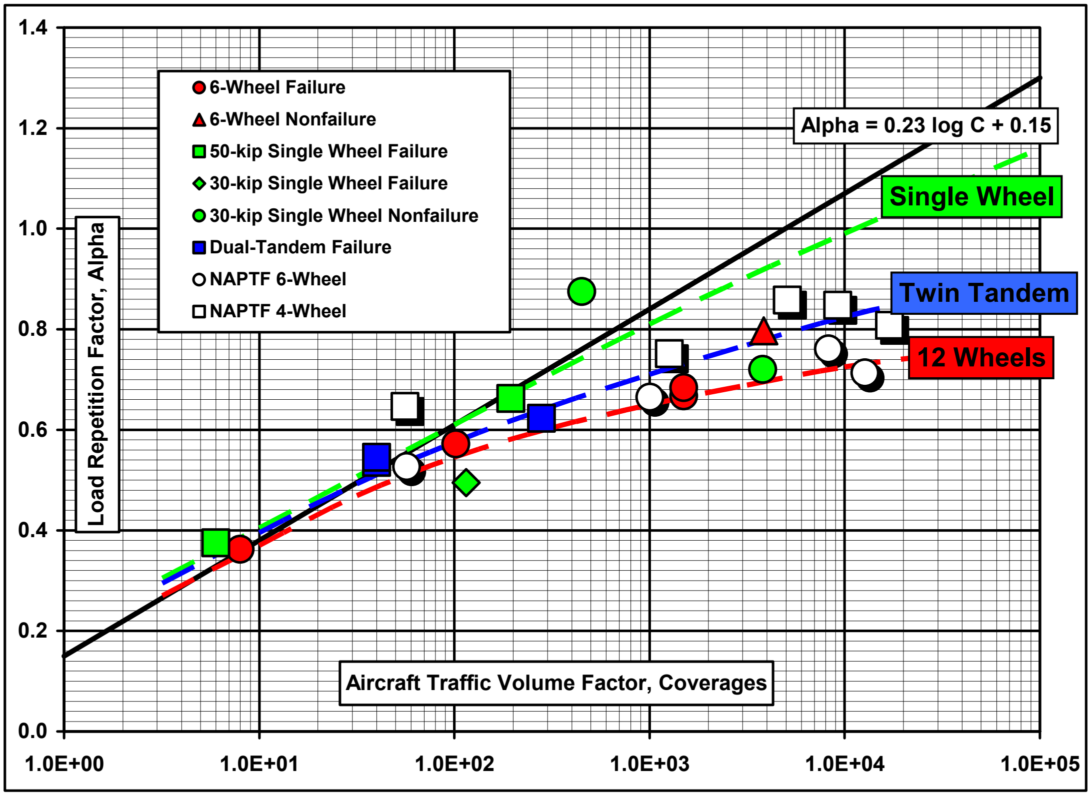

Figure 6. NAPTF and MWHGL alpha factor results with C5-A as a six-wheel gear and with conversion of NAPTF structures to equivalent MWHGL reference structures.

CRACK DATA

DATE	NO.	DIMENSIONS, IN A	B
3/18/02	1	49.50	63.50
	2	60.00	105.90
3/21/02	3	53.00	54.00
	4	42.65	43.25
3/22/02	5	53.00	102.00
	6	50.50	67.50
	7	126.00	134.90
	8	L	146.00
3/25/02	9	L	27.00
	10	L	94.00
	11	L	75.80
3/26/02	12	48.00	73.00
	13	69.00	105.00
	14	53.00	61.00
	15	L	154.00
3/27/02	16	35.50	36.00
	17	L	26.00
	18	38.50	48.00
	19	L	148.00
3/28/02	20	L	40.00
	21	30.00	40.00
	22	55.00	83.50
3/29/02	23	L	240.00
	24	L	226.50
	25	L	180.00
	26	T	24.00
	27	T	10.00
	28	55.00	71.25
	29	L	134.00
	30	L	180.00
	31	40.00	43.00
4/01/02	32	39.00	49.00
	33	T	49.00
	34	L	136.50
	35	T	50.00
	36	L	120.00
4/02/02	37	T	60.00
	38	L	74.00
	39	47.00	56.50
	40	49.00	102.50
	41	T	42.00
	42	47.00	150.00
4/03/02	43	T	26.00
	44	L	90.00
	45	L	64.00
4/04/02	46	48.00	87.50
	47	L	144.00
	48	L	39

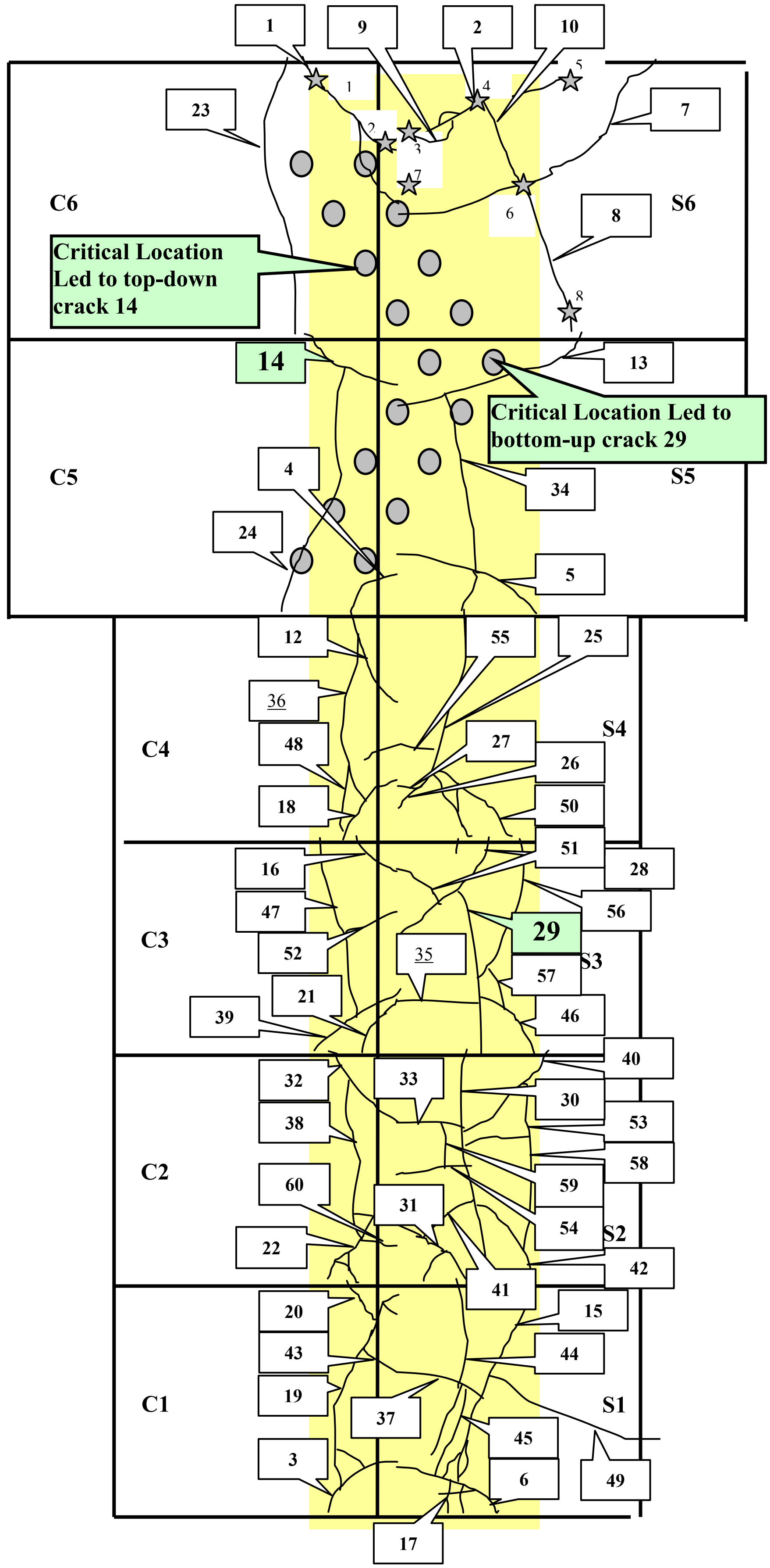

Figure 7. Cracks observed in CC2 test strip during trafficking.

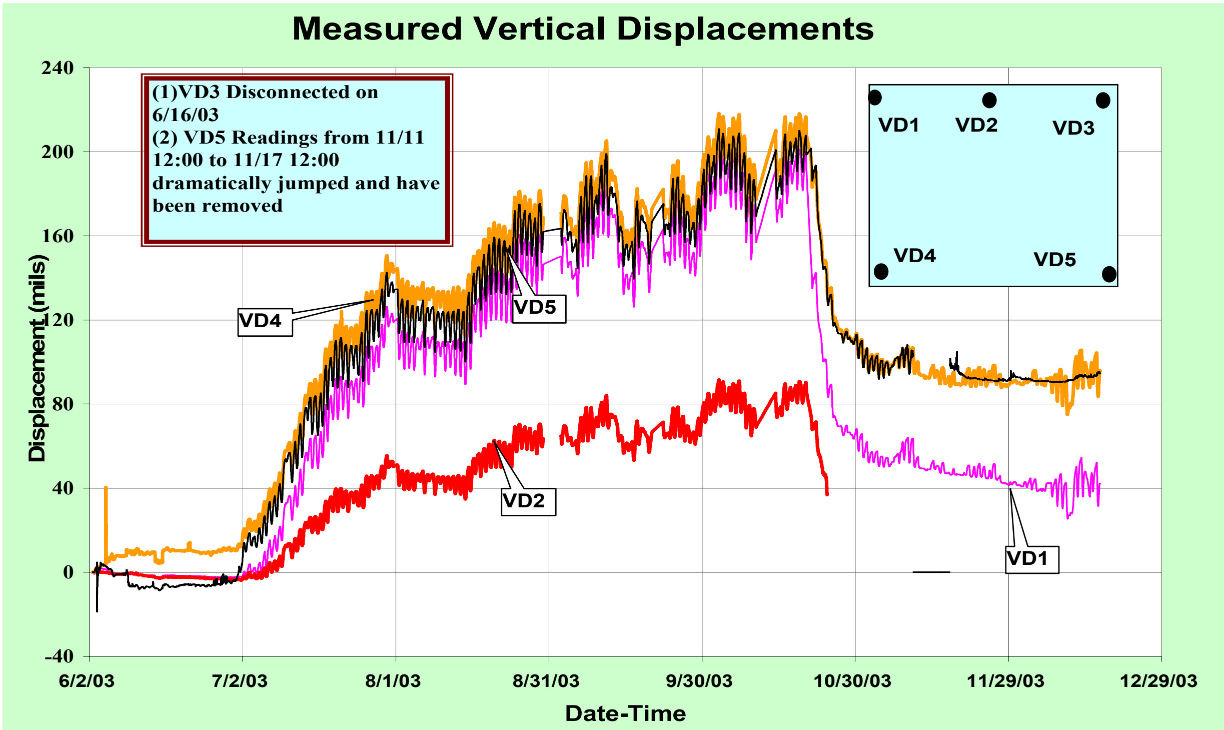

Figure 8. Vertical deflections measured at the corners and one edge of the free-standing slab.

Figure 9. Placing concrete for one of the CC2 test item slabs.

Figure 10. Strain gage installation for one of the CC2 test item slabs.